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Geotechnical Investigation

Main Sewage Pump Station Demolition and Site Improvements

Milpitas, California

Report No. 869-4 has been prepared for:

City of Milpitas

455 East Calaveras Boulevard, Milpitas, California 95305

June 24, 2004

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June 24, 2004
869-4

Mr. Tom Yousch & Ms. Lissette Morales
Swinerton Management & Consulting c/o
CITY OF MILPITAS
455 East Calaveras Boulevard
Milpitas, California 95305

**RE: GEOTECHNICAL INVESTIGATION
MAIN SEWAGE PUMP STATION
DEMOLITION AND SITE
IMROVEMENTS
MILPITAS, CALIFORNIA**

Dear Mr. Yousch and Ms. Morales

We are pleased to present the results of our geotechnical investigation for the above referenced development. Our report includes a description of the geotechnical and seismic aspects of the site along with our conclusions and geotechnical recommendations for design of the proposed Main Sewage Pump Station Demolition and Site Improvements.

We refer you to the text of the report for detailed recommendations. To help us continue to add value to your projects please visit the feedback section on our web site at www.lowney.com/feedback. Your opinion is important to us. If you have any questions concerning our findings, please call and we will be glad to discuss them with you.

Very truly yours,

LOWNEY ASSOCIATES

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MILPITAS, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the Main Sewage Pump Station Demolition and Site Improvements to be located in Milpitas, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the subsurface conditions at the site and to provide geotechnical recommendations for design of the proposed improvements.

For our use we received the following:

- Plans for "City of Milpitas, Main Sewage Pump Station Demolition and Site Improvements, 75% Complete Submittal Drawings" prepared by West Yost & Associates, not dated.

- Plans for "City of Milpitas, Main Sewage Pump Station Demolition and Site Improvements, 95% Complete Submittal Drawings," prepared by West Yost & Associates, not dated.

- Environmental Site Assessment for "Former Milpitas Sanitary District Treatment Plant, 1425 North McCarthy Boulevard, Milpitas, California," prepared by McCulley, Frick & Gilman, Inc., dated March 25, 1999.

1.1 Site History

The site is located along North McCarthy Boulevard in an agricultural and commercial area of the City of Milpitas.

According to the 1999 Phase I Environmental Site Assessment (ESA) performed by others (MFG, Inc, 1999), prior to its use as a treatment facility by the City of Milpitas, the site was originally owned by McCarthy Ranch and therefore, likely used for agricultural purposes. The sewage treatment facility was constructed in 1954, and included a digester, trickling filter, settling basin, operation building and a sludge lagoon. In 1964 the treatment plant was expanded to include several buildings and basins, and by 1970 had a total of five lagoons (MFG, Inc, 1999). In 1972, the City of Milpitas decommissioned the treatment plant. In 1974, the lift station on the south end of the property became operational and began pumping all sewage to the City of San Jose treatment plant (MFG, Inc, 1999). Since then, the decommissioned portion of the property has been used mainly for City storage. The adjacent Coyote Creek levee was realigned in the late 1980s.

Approximately 2,000 cubic yards of mounded soils exist along the west side of the concrete treatment basins. According Environmental Site Assessment Report prepared by McCulley, Frick & Gilman, Inc., dated March 25, 1999, the origin of the material is likely from a channel improvement project in 1990 to 1991. Undocumented mounded fill soils also exist along the northern portion of the property and just outside the eastern gate. The source of the northern fill material is unknown.

1.2 Project Description

As presently planned, we understand the project consists of two construction phases. Phase 1 will include of removal of the existing operations/control building, sedimentation basins, digester, settling basins, sludge basing, chlorine contact basin, chlorine building, trailer and shed structures, miscellaneous piping, control panels, fill materials and other miscellaneous stockpiled materials. Several structures scheduled for demolition extend to depths up to about 16 feet below grade. Demolition of the structures may include full or partial removal, depending on the particular situation. This is discussed in more detail later in this report. Re-use of on-site materials as backfill for the abandoned structures is discussed in this report. We are also working with the project civil engineer on preparation of demolition specifications for incorporation into the project specifications. Phase 1 will also include drainage and temporary paving of the corporation yard.

We understand that the northern boundary of the proposed staging yard for the City will end approximately 260 feet south of the current northern boundary of the site. This land will go back to the McCarthy Ranch owner.

Phase 2, beginning next year, will include permanent paving, landscaping, permanent storm water measures, and construction of the operations building. At this time, the location of the building has not been determined.

1.3 Scope of Services

Our scope of services was presented in detail in our agreement with you dated March 29, 2004. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling six borings and retrieving soil samples for observation and laboratory testing, advancing two Cone Penetration Tests (CPTs), excavating six test pits, and collecting several bulk samples.
- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples. Correlation of CPT interpretations with visual classification and laboratory testing on samples collected from our borings.
- Review of aerial photographs showing site history.
- Engineering analysis to evaluate site earthwork, demolition, building foundations, and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

We also provided environmental services for this project and provided our conclusions and recommendations under a separate cover.

2.0 SITE CONDITIONS

2.1 Geology

The site is located on alluvial deposits within the northwestern portion of the Santa Clara Valley, approximately 2 miles southeast of San Francisco Bay. Bishop and Williams (1974b) show alluvial deposits as being 500 feet deep beneath the site. Geologic mapping by Helley and Brabb, 1971, indicate the site is located on younger alluvial fan deposits consisting of organic-rich clay and silty clay. The valley alluvium is of Pleistocene to Holocene age, and has been deposited principally as a series of coalescing alluvial fans. The deposits in these alluvial fans become progressively finer-grained towards San Francisco Bay. The site is mapped as being within a liquefaction hazard zone by the State of California (CGS, 2001 - Milpitas Quadrangle).

2.2 Exploration Program

Subsurface exploration was performed on May 20, 2004 using four-wheel rubber-tired backhoe equipment to investigate and sample the subsurface soils. Six exploratory test pits were excavated to depths ranging from 4½ to 7 feet.

Subsurface exploration was also performed on May 26 and 28, 2004, using conventional, truck-mounted hollow-stem drilling and Cone Penetration (CPT) equipment to investigate, sample, and log subsurface soils. Six exploratory borings and two CPTs were advanced to depths ranging from 20 to 45 feet. Our borings and CPTs were permitted and backfilled in accordance with Santa Clara Valley Water District and the City of Milpitas guidelines. Representative bulk samples of the surface soil from Boring EB-1 and EB-2 were obtained for pavement design purposes. Four representative bulk samples were also obtained from the berm located parallel to the east fence of sewage plant station and sent to the Soil and Plant Laboratory.

The approximate locations of the borings, CPTs, test pits, and bulk samples are shown on the Site Plan, Figure 2. Logs of our borings and CPTs, and details regarding our field investigation are included in Appendix A; our laboratory tests are discussed in Appendix B.

2.3 Surface

We also performed a brief surface reconnaissance during our site exploration. The facility is surrounded on all sides by chain link fence. Access into the site through the chain link fence is in the south end of site. It appears that there was a northern entrance that has not been maintained. The site is bordered by North McCarthy Boulevard to the east, a flood control levee with an asphalt concrete biking trail on top and Coyote Creek to the west, and open fields to the north and south. A landscaping berm is located to the east of the site between the fence and North McCarthy Boulevard. As discussed above, stockpiled soil occurs in a berm along the west side of the settling/sludge/ chlorine contact basin and chlorine basin, and undocumented fills occur in the northern part of the site.

Based on the topographic information provided by West Yost & Associates, the project civil engineers, the site is relatively level inside the fenced sewage pump station with approximately three feet of topographic relief between Elevations 8 and 11 feet, except for the berms or stockpiles on the site. The approximately 80 feet by 40 feet berm southwest of the digester is approximately 9 feet high, the 400 feet by 30 feet stockpile parallel to the sludge basin on the west side is approximately 4½ feet above adjacent site grades and is being used as an access road to the north end of the site, and the stockpiles covered with brush in the north end of the site extend about 160 feet south of the north chain link fence and are approximately 6 feet above existing site grades. The flood control levee, which continues along the west side of the site, slopes down to the north from about Elevation 17½ feet near the south property boundary to about 14 feet at the north end of the site (about 4 to 8 feet over the average site grades). The landscaping berm, which extends along the full length to the east of the site between the fence and McCarthy Boulevard is approximately 4 feet high and includes mature trees.

The road to the east of the sludge basin leading to the north end of the site appeared to be paved. A gravel access road was at the entrance of the site in the south end; the majority of the site was unimproved. We did not drill through the eastern paved road and therefore did not observe the structural section. The location of existing structures and stockpiles can be seen in the Site Plan, Figure 2.

2.4 Subsurface

Generally, the site is blanketed with about 5 to 7 feet of low to moderately plastic native silty and sandy clays over loose sand with varying amounts of silt or sandy silt to depths of about 10 to 12 feet. Underlying the surficial soils, a 4- to 10-foot-thick soft, highly plastic clay layer with variable amounts of organics was encountered to a depth of about 17 to 22 feet. The shallow sand layer and soft clay layer were not encountered in CPT-2, advanced in the southern portion of the property. Below the high plasticity clay layer, stiff lean clays with interbedded layers of medium dense to dense sand was encountered to a depth of 45 feet, the maximum depth explored.

Based on the locations of the sludge lagoons and ponds from Figure 2 of the Environmental Site Assessment Report by McCulley, Frick & Gilman, Inc., dated March 25, 1999, Borings EB-3 through EB-6 were located to observe the conditions of the sludge lagoon fills. After visually observing the soil samples it appears Borings EB-3 was drilled at the southern edge of the lagoon and EB-6 was drilled outside the location of the sludge ponds. Boring EB-3 encountered about 4 feet of fill consisting of hard lean clay. Boring EB-4 was drilled in the 1965 – 1970 sludge lagoon and Boring EB-5 was drilled in Pond #1 (1970 - 1989). Both borings encountered fill to about 8 to 10 feet below existing site grade. The fill consisted of very stiff to hard lean clay to about 4 to 6 feet over stiff to hard silt with sand. Organics were observed at around 10 feet.

Test Pits TP-1 and TP-2 were excavated in the fill berm west of the sludge basin. The berm material consisted primarily of varying amounts of silt, sand, and clay to the excavated depth of five feet. Some gravel, wood, concrete and asphalt chunks, and geofabric was noted in the top 2 feet of the berm. Test Pit TP-3 was performed in a 5-foot-high mound consisting of silty soil with organics, wood, plastic, rebar, and gravel debris. Test Pits TP- 4, TP-5, and TP-6 were generally free of debris except in the top foot with asphalt chunks, gravel, or aggregate base. Sandy silt fill material was encountered in Test Pits TP- 4, TP-5, and TP-6 to approximately 3½ feet below site grade. Below the fill, native material was encountered to 7 feet, the maximum depth explored using a backhoe. The native material consisted of silt and sand.

2.4.1 Laboratory Testing

A Plasticity Index (PI) test was performed on a representative sample of the surficial native material from Boring EB-1. The test resulted in a PI of 13 indicating low expansion potential. Percent organic tests were performed on samples collected from the sludge lagoon fills. The results indicate organic contents on the order of 2 to 3 percent.

The percent organic test results from the highly plastic clay layer range up to 15 percent, possibly indicating that this clay is a Bay Mud layer. Any excavations extending to approximately 10 feet will encounter the highly plastic clay layer, which has in-situ moisture contents approximately 15 to 45 percent over estimated laboratory optimum moisture. Due to high in-situ moisture content special precautions should be taken when exposing and working over this material.

We also submitted five bulk samples to the Soil and Plant Lab to evaluate materials for landscape soil. Four samples were collected from the test pits within the berm along North McCarthy Boulevard and one was collected from Test Pit TP-3, one of the on-site stockpiles. The results from the testing and their amendment recommendations are attached in Appendix B.

2.5 Ground Water

Free ground water was encountered during subsurface exploration in exploratory Borings EB-1 at a depth of 17 feet; pore pressure measurements taken in CPT-1 indicated depth to ground water at 15 feet. In addition, the ground water level was recorded at a depth of 9½ and 10½ feet in Borings EB-4 and EB-5, respectively, approximately 5 hours after drilling. Please note the ground water depth measurements were taken at the time of drilling for all projects reviewed as well as the current borings and may not reflect a stabilized level. Based on our previous explorations in the area and State mapping (CGS, 2001 - Milpitas Quadrangle), ground water levels in the vicinity are known to be at depths on the order of 5 feet from the average site grade. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture Hazard

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. Table 1 lists the faults within 25 kilometers of the site:

Table 1. Approximate Distance to Seismic Sources

Fault	Distance (miles)	Distance (kilometers)
Hayward (Southeast Extension)	1.9	3.1
Hayward (Total Length)	4.2	6.7
Calaveras	6.7	10.8
Monte Vista - Shannon	12.4	20.0

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a State of California Alquist-Priolo Fault Zone (California Department of Mines and Geology (CDMG), 1982) for fault rupture or located within a Santa Clara County Geologic Hazard Zone for fault rupture (Santa Clara County, (2002). Fault rupture through the site, therefore, is not anticipated. The site is also not located within a State of California Seismic Hazard Zone for earthquake-induced landslides (CGS, 2001) or within a City of San Jose Special Studies Zone for earthquake hazards (1983). However the site is located within the seismic hazard zone for earthquake-induced liquefaction (CGS, 2001) and within a Santa Clara County Geologic Hazard Zone for liquefaction (Santa Clara County, (2002), which is discussed in the "Liquefaction" section.

3.1.1 Maximum Estimated Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to virtually all developments in the San Francisco Bay Area. The Probabilistic Seismic Hazard Analysis (PSHA) performed by the California Geologic Survey, estimates a peak horizontal ground acceleration of 0.68g at the site with a 10 percent probability of exceedance in 50 years. This ground acceleration is also used in our liquefaction analysis.

3.1.2 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2002), referred to as WG02, determined there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2003 and 2032. This result is an important outcome of WG02's work, because any major earthquake can cause damage throughout the region.

The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.2 Liquefaction

The site is located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards (CGS, 2001 - Milpitas Quadrangle) and in a Santa Clara County Geologic Hazard Zone (Santa Clara County, 2002) mapped liquefaction zone. During cyclic ground shaking, such as during earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix, resulting in liquefaction. Liquefied soil may loose shear strength that may lead to large shear deformations and/or flow failure under moderate to high shear stresses, such as beneath foundations or sloping ground (NCEER/NSF, 1998), and in many ways may behave more like a liquid than a solid. Liquefied soil can also settle (compact) as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, in some cases, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured.

Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

3.2.1 Analysis and Results

As noted in the subsurface description above, several sand and silt layers were encountered below the recommended design ground water depth of 5 feet. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed operations building.

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd, et al., 2001) in accordance with guidelines set forth in CDMG Special Publication 117 (CDMG, 1997). The NCEER methods for SPT and CPT analyses update simplified procedures presented by Seed and Idriss (1971).

In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. CRR calculations are based on CPT tip resistance. As we utilized hollow-stem drilling methods for the borings, only the data from the CPTs was used for the liquefaction analysis. To account for effective overburden stresses and soil behavior, the CPT tip pressures were corrected for overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how plastic the soil behaves.

The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65(\frac{a_{max}}{g})(\frac{\sigma_{vo}}{\sigma'_{vo}})r_d$$

where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We use a probabilistic horizontal acceleration of 0.68g, corresponding to a 10 percent chance of exceedance in 50 years. Based on published maps by the CGS, a 7.1Mw event on the Hayward Fault most contributes to the probabilistic peak ground acceleration at the site.

Soils that have significant amounts of plastic fines (greater than about 25 percent) or an I_C greater than 2.6, and soils with corrected CPT tip resistances greater than 160 are considered either too plastic or too dense to liquefy. Such soil layers have been screened out during our analysis and are not presented below.

The FS against liquefaction can be expressed as the ratio of the cyclic resistance ratio (CRR) to the cyclic stress ratio (CSR). If the FS for a soil layer is less than 1.0, it is possible that the soil layer may liquefy during a moderate to large seismic event.

$$FS = \frac{CRR}{CSR}$$

A summary of our analysis for CPT data are presented in the tables below. An analysis was not performed on the SPT data collected in hollow stem borings, since blow counts in hollow stem borings may be unreliable in sands below the ground water table.

Table 2. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	I_C	$*q_{C1N}$	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)	Estimated Differential Settlement (in.)
CPT-1	5.1	11.0	2.1	80.9	0.2	Liquefaction Likely	3.6	1.8
CPT-1	36.2	0.6	1.9	97.8	0.2	Liquefaction Likely	0.2	0.1
						Total =	**3.8**	**1.9**
CPT-2	5.1	0.8	2.5	75.2	0.2	Liquefaction Likely	0.5	0.3
CPT-2	6.7	1.1	2.5	63.9	0.2	Liquefaction Likely	0.6	0.3
CPT-2	10.3	<0.2	2.6	64.1	0.2	Liquefaction Likely	<0.1	<0.1
CPT-2	24.1	<0.2	2.6	116.3	0.3	Liquefaction Likely	<0.1	<0.1
						Total =	**1.2**	**0.6**

* CPT tip pressure corrected for overburden and fines content

Our analyses indicate that several silt and sand layers theoretically can liquefy, resulting in up to about 3.8 inches of total settlement, and 1.9 inches of differential settlement, based on confined liquefiable layer methods. Estimates of volumetric change and settlement were determined by the Ishihara and Yoshimine (1990) method. As discussed in the SCEC report, differential movement for level ground, deep soil sites, will be on the order of half the total estimated settlement.

The methods of analysis used to determine estimated total settlement do not take into account the possibility of surface ground rupture. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer. Based on work by Youd and Garris (1995), it is our opinion that there is not enough of a cap of non-liquefiable material to prevent ground rupture at the site due to liquefaction of the shallow sand layers encountered in CPT-1 and Borings EB-1, EB-2, EB-3, and EB-6. If ground rupture and sand venting occur, significantly higher ground deformation could occur. As the potentially liquefiable layers encountered in CPT-2 are relatively thin, surface venting is not anticipated in this area. A further discussion is in the Conclusions section of this report.

3.3 Seismically-Induced Settlements

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform settlement of non-saturated soil strata, resulting in movement of the near-surface soils. We judge the probability of significant differential compaction at the site to be moderate to high for improvements that span fill (the area of the previous lagoons) and undisturbed native materials. A further discussion is in the Conclusions section of this report.

3.4 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free. Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur.

The site is separated from Coyote Creek by a 3- to 7-feet-high flood control levee from existing site grade. The proposed revised northern property boundary will be on the order of 300 feet, the closest location of Coyote Creek, which is beyond a distance for lateral spreading to affect site improvements. For these reasons, the probability of lateral spreading occurring at the site during a seismic event is low.

3.5 Flooding

Flooding may be caused by intensive rainfall, tsunami or seiche, or dam or levee breaks. The terms tsunami and seiche describe ocean tidal waves or similar waves in closed bodies of water.

Intensive Rainfall: As shown on the July 4, 1988 Federal Emergency Management Agency (FEMA) "Flood Insurance Rate Map" (FIRM), this site is within Zone A. Zone A is described as "Areas 100-year flood; base flood elevations and flood hazard factors not determined."

Tsunami or Seiche: The site is several miles inland from the San Francisco Bay shore (water front). Hence, the potential for inundation due to tsunami and/or seiche is considered remote.

Dam Break: In the March 1982 Santa Clara County General Plan, 12 dams were identified. None of these dams were expected to inundate the Milpitas area if a failure occurred. The current Santa Clara County General Plan (1994) does not identify areas potentially subject to inundation due to dam failures that may occur suddenly due to an earthquake or dam overtopping during periods of intense precipitation. Dams in the County periodically undergo dam safety inspections by the State's Division of Safety of Dams. In addition, strengthening and modifications to dams and spillways to provide structural safety of the reservoirs in Santa Clara County are ongoing efforts of the Santa Clara County Water District. For these reasons, we judge inundation due to dam failure to be a low risk for the site.

4.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

4.1 Conclusions

From a geotechnical engineering viewpoint the proposed development may be constructed as planned, in our opinion, provided the design is performed in accordance with the recommendations presented in this report. The primary geotechnical concerns at the site are related choosing the location of the future operations building and the demolition of the current structures.

Building Location Concerns

- The potential for seismic settlement ground surface disruption
- Differential movement due to spanning native/fill transitions
- Soft compressible clay layer

Demolition Concerns

- The re-use of on-site fill and/or soil stockpiles
- The presence of shallow ground water
- The proximity of the sedimentation basin to the flood control levee

For this report, we have prepared a brief description of the issues and presented typical approaches to manage potential concerns associated with the long-term performance of the development.

4.1.1 The Potential for Seismic Settlement and Ground Surface Disruption

Our analyses indicate that several silt and sand layers theoretically can liquefy, resulting in significant total settlement, including the potential for surface venting, which would increase the total settlement. The shallow layers encountered where this could occur were noted in the areas of CPT-1 and Borings EB-1, EB-2, EB-3, and EB-6. We don't anticipate ground rupture in the area of CPT-2. This potential for significant settlement and ground rupture should be considered as locations for the future operations building are discussed. If the building will be located where ground rupture is anticipated, the two options are ground improvement to support shallow foundations or supporting the structure on deep foundations, estimated to be on the order of 30 to 50 feet long. Ground improvement may consist of earthwork removal and densification of the sand layers, stone columns, soil-cement improvement of the sand layers, or other methods. If deep foundations are chosen, consideration should be given to the slab-on-grade support. If ground rupture occurs, major cracking of the slab could require replacement of the slab. Alternatively, a structural slab constructed over a 12-inch void form could limit the amount of cracking.

4.1.2 Differential Movement due to Spanning Native/Fill Transitions

In addition to the liquefaction potential, if the building location is in the north half of the site then, it is likely that the building footprint will span both native and fill material used to backfill the sludge lagoons. If the ground improvement discussed above occurs, supporting the building on grid foundations or a mat foundation capable of withstanding some differential settlement would be possible. Supporting the structure on deep foundations would also mitigate the potential for differential movement.

4.1.3 Soft Compressible Clay Layer

The soft, highly plastic clay layer encountered at a depth of 10 to 12 feet was about 4- to 10-feet-thick. Based on review of the CPT logs, it appears that this layer does not extend to CPT-2 in the southern portion of the site. This layer could consolidate under the proposed building loads, causing some additional minor settlement. If the ground improvement discussed above occurs, supporting the building on grid foundations or a mat foundation capable of withstanding some differential settlement would be possible. Supporting the structure on deep foundations would also mitigate the potential for total and differential movement within the clay layer.

4.1.4 The Re-Use of On-Site Fill and/or Soil Stockpiles

We understand that the City is considering re-use of the soil from the onsite stockpiles and the berm west of the settling tanks as backfill material for the demolished structures. The berm west of the settling tanks appears to be suitable for fill provided the debris encountered in the top two feet is removed, according to the "Material for Fill" section of this report. Organic content testing was performed on a soil samples from Test Pits TP-3 and TP-5. The test results indicate that the onsite stockpiles of material in the northern end of the site have about 2 to 4 percent organic matter. Due to the variable quantities of organic material and surficial tall weeds, separation of suitable portions of the stockpiled materials may not be feasible. We recommend that consideration be given to removal of the northern stockpiles.

Please refer to our Phase 2 environmental assessment report (prepared under separate cover) for any special recommendations concerning re-use of on-site fill material or off-haul.

4.1.5 The Presence of Shallow Ground Water

Full demolition and removal of the sedimentation basins, digester, settling basins, sludge basins, and chlorine contact basin extend near or below the ground water table. As discussed at project meetings, with the exception of the digester, the structures will be removed to a depth of 5 to 7 feet below grade. This will include removal of structure walls and creating pressure relief holes in the slabs prior to backfilling. We understand that the digester will be fully removed. This structure extends about 15 to 16 feet below grade. Contractors should plan on dewatering and subgrade stabilization where the soft clay layer is encountered. We understand that at this time, the City does not want to use crushed rock in the bottom of any excavations; therefore the ground water must be pumped down for the demolition in order to backfill. Sand cement slurry may also be used at the bottom of an excavation to provide a stable surface on which to backfill. Consideration may be given to leaving the settling basin slab, which extends about 3 feet below grade, in place if it is not in conflict with future improvements.

4.1.6 The Proximity of the Sedimentation Basin to the Flood Control Levee

The sedimentation basin is located adjacent to the flood control levee. Demolition of the structure walls to 5 to 7 feet below grade could de-stabilize the levee. We recommend that the western wall remain in place and that the backfill occur up to at least 5 feet below grade prior to removal of the other walls and completion of the backfill.

4.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and test the geotechnical aspects of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation, and if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

5.0 EARTHWORK

5.1 Clearing and Site Preparation

The site should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, fills, pavements, debris, designated trees, shrubs, and associated roots.

Abandonment of existing buried utilities is discussed below. We recommend that trees and shrubs designated to be removed should include the entire rootball and all roots larger that ½-inch in diameter. Depressions resulting from removal of trees and shrubs should be cleaned of loose soils and roots, and properly backfilled in accordance with the "Compaction" section of this report. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of buried structures be carried out under our observation and that backfill be tested during placement.

After clearing, any vegetated areas should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. At the time of our field investigation, we estimated that a stripping depth of approximately 4 inches would be required in vegetated areas. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

5.1.1 Demolition – Below-Grade Structures

Full demolition and removal of the sedimentation basins, digester, settling basins, sludge basins, and chlorine contact basin extend near or below the ground water table. As discussed at project meetings, with the exception of the digester, the structures will be removed to a depth of 5 to 7 feet below grade. This will include removal of any sludge or water contained within the structures, removal of the structure walls, and creating pressure relief holes in the slabs prior to backfilling. Side slopes of fill excavations above the depth of structural removal should be sloped at inclinations no greater than 3:1 (horizontal to vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

We understand that the digester will be fully removed. This structure extends about 15 to 16 feet below grade. Contractors should plan on dewatering and subgrade stabilization where the soft clay layer is encountered. We understand that at this time, the City does not want to use crushed rock in the bottom of any excavations; therefore the ground water must be pumped down for the demolition in order to backfill. Sand cement slurry may also be used at the bottom of an excavation to provide a stable surface on which to backfill.

Consideration may be given to leaving the settling basin slab, most of which extends about 3 feet below grade, in place if it is not in conflict with future improvements.

The sedimentation basin is located adjacent to the flood control levee. Demolition of the structure walls to 5 to 7 feet below grade could de-stabilize the levee. We recommend that the western wall permanently remain in place and that the interior backfill occur up to at least 5 feet below grade prior to removal of the other walls and completion of the backfill. The contractor should evaluate the need for any internal bracing to complete the demolition and backfill operation of this structure.

Please refer to our Phase 2 environmental assessment report for recommendations on how to dispose of the sludge and water removed from within the structures. The sludge may not be used as backfill material from a geotechnical viewpoint.

5.1.2 Demolition – At-Grade Structures

The slabs and footings of all at-grade structures scheduled for demolition should be completely removed. Side slopes of fill excavations should be sloped at inclinations no greater than 3:1 (horizontal to vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

5.2 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. As an alternative it may be feasible to abandon (in-place) underground utilities within the proposed building area provided the utility does not conflict with new improvements, is completely grouted, and previous fills associated with the utility do not pose a risk to the structure. Existing underground utilities outside the proposed building area(s) should be removed or abandoned in-place by grouting or plugging the ends with concrete. The decision to abandon in-place versus removal should be based on the level of risk associated with the particular utility line.

It should be noted that fills associated with underground utilities abandoned in-place may have an increased potential for settlement, and partially grouted or plugged pipelines will have a potential risk of collapse that may result in ground settlement, soil piping, and leakage of pipeline constituents. The potential risks are relatively low for small diameter pipes (4 inches or less) and increasingly higher with increasing diameter.

5.3 Subgrade Preparation

The following applies to where excavation bottoms are in soil or other surficial areas where surface improvements are planned. After the site has been properly cleared, stripped, and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs-on-grade, foundations, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section.

The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment. If the relative compaction of the subgrade is less than recommended or the surface of the subgrade has significant yielding, then the area should be over-excavated and rebuilt or reworked until the subgrade conforms to our recommendations.

5.4 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. As discussed above, we do not recommend re-use of the materials stockpiled in the northern portion of the site. The fills in the berm along the west side of the settling and sludge basins may be re-used. Please note that isolated chucks of asphalt and pieces of geofabric were noted in the test pits. Oversized debris and fabric should be removed. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Imported fill material should be inorganic and should have a Plasticity Index of 15 or less. Non-expansive fill material should be inorganic and should have a Plasticity Index of 15 or less. Imported fill should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of proposed import fill should be submitted to us at least 10 days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations. It should be noted, that laboratory testing can take up to 10 days to complete.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides, and resistivity.

5.5 Compaction

All fill, as well as scarified surface soils in those areas to receive fill or slabs-on-grade, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum, except for the native expansive clays. If the native expansive clays are encountered they should be compacted to between 87 and 92 percent relative compaction at a moisture content at least 3 percent over optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition), except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

5.6 Wet Weather Conditions

It should be understood that earthwork such as fill placement and trench backfill may be very difficult during wet weather, especially for fill materials with a significant amount of clay. If the percent water in the fill increases significantly above the optimum moisture content, the soils will become soft, yielding, and difficult to compact. Therefore, we recommend that earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

As discussed previously, the native fat clay encountered approximately 10 feet below site grade generally has in-situ moisture contents 15 to 45 over estimated laboratory optimum. Contractors should plan on drying of the soils prior to reuse as engineered fill.

5.6.1 Chemical Treatment

We understand that the City plans to chemically treat the pavement subgrade, which is discussed in the "Pavement" section of this report. If wet subgrade or import material cannot be aerated and compacted within the project schedule time allotment, chemical treatment can be used to dry out the materials. Our experience with stabilizing the McCarthy Commercial Center to the south indicates that 3½ to 4 percent Quicklime or 4 to 5 percent contractors blend "KD83" (a quicklime/cement kiln dust blend) will most likely be suitable for treatment of both the subgrade. If permanent improvement of the R-value is desired, we estimate that 4 percent Quicklime or 5½ percent KD83 applied to the upper 12 inches of pavement subgrade will provide stabilization of the soil and at least an R-value of 50. These percentages should be used for estimating purposes only at this time. A lime study and an R-value test of the treated material should be performed to verify the percentage of chemical additive and the design R-value.

5.7 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas and coming into contact with expansive subgrade material.

5.8 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

5.9 Temporary Dewatering

As previously discussed, measured ground water elevations and historic high ground water levels are above the planned excavation depths for some structures; therefore, temporary dewatering will be necessary during construction. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions at our site, they should be aware that modifications to the dewatering system, such as adding well points, may be required during construction depending on the conditions encountered.

We recommend that any dewatering of the site be carried out in such a manner as to maintain the ground water a minimum of 5 feet below the bottom of the mass excavations. The contractor should design a system to achieve this criteria. Should dewatering be temporarily shut down, it could have considerable detrimental affects on the excavations, including flooding, destabilization of the bottom of the excavation, shoring failures, etc. Therefore, we recommend that consideration be given to having the dewatering contractor provide backup power in case of loss of power or other redundancies, as deemed necessary.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge at an offsite facility.

5.10 Surface Drainage

Positive surface water drainage gradients (2% minimum) should be provided adjacent to the structures to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundations and slabs-on-grade. The construction paving section should be sloped to provide surface drainage to suitable facilities.

5.11 Construction Observation

A representative from our company should observe and test the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including, site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

6.0 FOUNDATIONS

As discussed in the "Conclusions" section, there are several concerns with respect to locating the proposed operations building in the northern two-thirds of the site. The primary concern is the potential for significant liquefaction-induced settlements, including the potential for ground rupture. There is also the potential for differential foundation movement across native/fill transitions and consolidation of the soft clay layer. The mitigation options include ground modification to support shallow foundations or supporting the structure on deep foundations, extending on the order of 30 to 50 feet. Ground improvement may consist of earthwork removal and densification of the sand layers, stone columns, soil-cement improvement of the sand layers, or other methods. If deep foundations are chosen, consideration should be given to the slab-on-grade support. If ground rupture occurs, major cracking of the slab could require replacement of the slab. Alternatively, a structural slab constructed over a 12-inch void form could limit the amount of cracking.

The explorations performed to date should be used to help site the building. Further subsurface exploration should be performed once building locations have been determined. The additional subsurface data will be used to provide design-level recommendations for construction of the building. Please note that the main geotechnical concerns discussed above do not occur in the area of CPT-2.

6.1 1997 UBC/2001 CBC Site Coefficients

Chapter 16 of the 1997 Uniform Building Code and 2001 California Building Code describes the procedure for seismic design of the structure, which includes the Seismic Coefficients C_a and C_v. These coefficients are developed from parameters contained a series of tables and figures in the code, listed in Table 3 below. Section 1636.2 and Table 16-J describes the procedure for assigning a soil profile type (S_A through S_F) to the site. Based on our borings, the site is underlain by liquefiable soils subject to surface venting. Therefore, site can be characterized as soil profile type S_E generally described as a soft soil profile with average Standard Penetration Test (N) values less than 15 blows per foot. Based on the above information, the site may be characterized for design based on Chapter 16 of the 2001 CBC using the information in Table 3 below. Please note that the controlling Seismic Source listed below may not be the nearest to the site.

Table 3. 1997 UBC/2001 CBC Site Categorization and Site Coefficients

Categorization/Coefficient	Design Value
Soil Profile Type (Table 16-J)	S_E
Seismic Zone (Figure 16-2)	4
Seismic Zone Factor (Table 16-I)	0.4
Seismic Source Name	Hayward Total/ Hayward Southest
Seismic Source Type (Table 16-U)	A/ B
Distance to Seismic Source (kilometers)	6.7/ 3.1
*Near Source Factor N_a (Table 16-S)	1.19
Near Source Factor N_v (Table 16-T)	1.46
Seismic Coefficient C_a (Table 16-Q)	0.43
Seismic Coefficient C_v (Table 16-R)	1.41

*Note: For Seismic Zone 4, the near-source factor Na used to determine Ca need not exceed 1.1 for structures complying with all the conditions within UBC Section 1629.4.2.

7.0 PAVEMENTS

7.1 Chemical Treatment and Asphalt Concrete Pavements

Based on R-value testing and our experience in the vicinity, the R-value of the native subgrade soil ranges from 5 to 20. Due to the variability, we judge an un-treated design R-value of 5 to be appropriate. As discussed in previous meeting and communications with the City of Milpitas, it is our understanding that the City would like to chemically treat the subgrade at the main sewage pump station. Chemical treatment will mitigate pumping subgrade issues that could arise during construction and allow the City to proceed with construction in the winter months. The treatment will also provide a more uniform subgrade R-value.

Our experience with stabilizing the McCarthy Commercial Center to the south indicates that 3½ to 4 percent Quicklime or 4 to 5 percent contractors blend "KD83" (a quicklime/cement kiln dust blend) will most likely be suitable for stabilization of the subgrade. If permanent improvement of the R-value is desired, we estimate that 4 percent Quicklime or 5½ percent KD83 applied to the upper 12 inches of pavement subgrade will provide stabilization of the soil and at least an R-value of 50. These percentages should be used for estimating purposes only at this time. A lime study and an R-value test of the treated material should be performed to verify the percentage of chemical additive and the design R-value. The lime study and R-value testing will need to be performed at least two weeks prior to construction.

Recommendations for pavement sections constructed over chemically-treated subgrade with an R-value of 50 are presented below. Using a Traffic Index (TI) of 9.0 provided by the City, we developed the following recommended pavement section based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 4. The permanent asphalt concrete pavement section should consist of 5.5 inches of asphalt concrete over 6.5 inches of Class 2 aggregate base.

The initial/interim construction structural section should consist of at least 6 to 8 inches of Class 2 aggregate base overlying the chemically-treated subgrade. The subgrade and aggregate base should be sloped to drain. Prior to construction of the final pavement section, the upper 2 to 3 inches of aggregate that has become contaminated with dirt over the winter will need to be removed.

7.2 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

7.3 Exterior Sidewalks

We recommend that exterior concrete sidewalks be at least 4 inches thick and underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Portland Cement Concrete Pavements" section of this report.

8.0 LIMITATIONS

This report has been prepared for the sole use of the City of Milpitas, specifically for design of the Main Sewage Pump Station Demolition and Site Improvements in Milpitas, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discreet locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Lowney Associates cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Lowney Associates' report by others. Furthermore, Lowney Associates will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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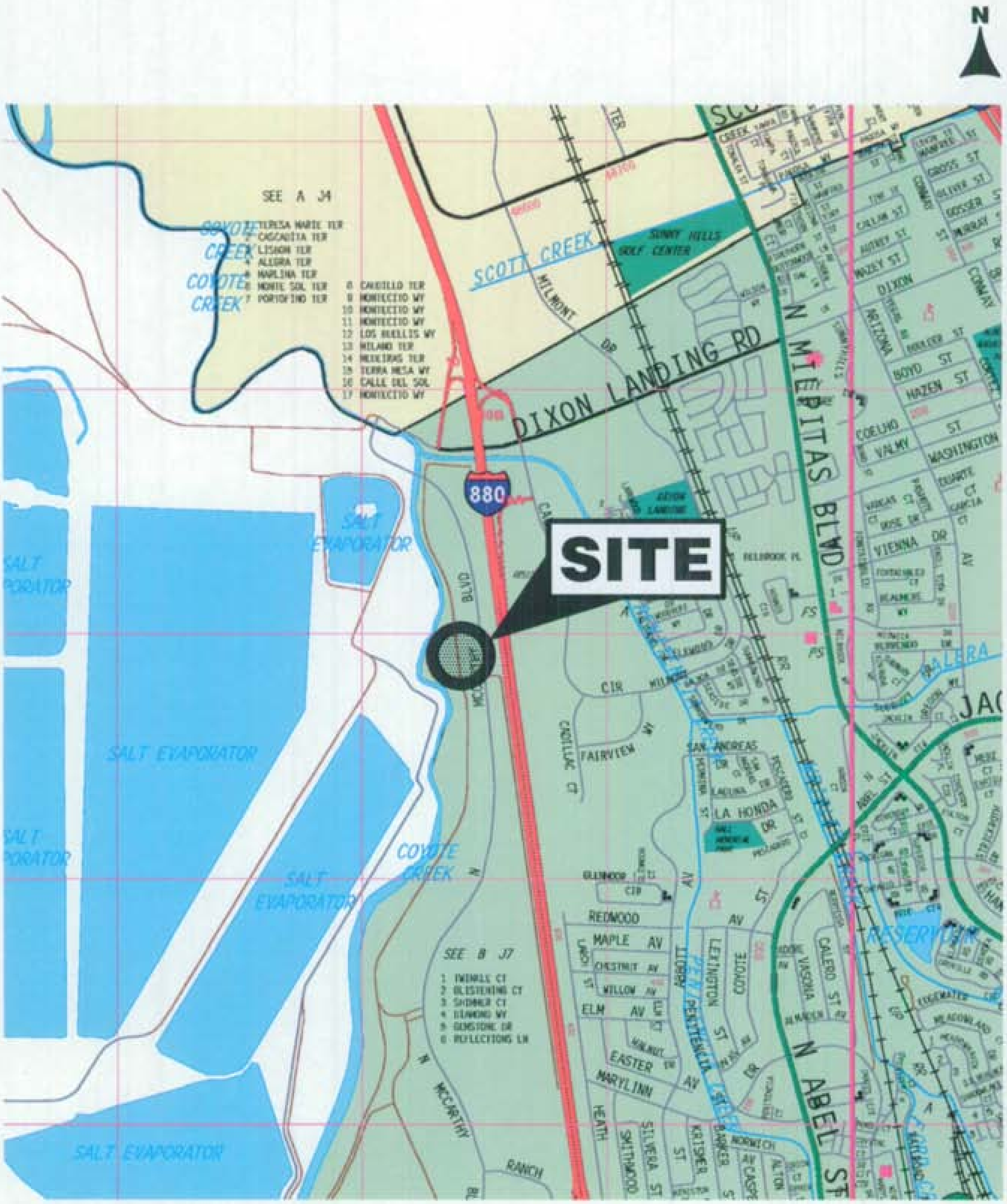

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6/04'EB

VICINITY MAP

MAIN SEWER PUMP STATION

Milpitas, California

LOWNEYASSOCIATES

Environmental/Geotechnical/Engineering Services

FIGURE 1

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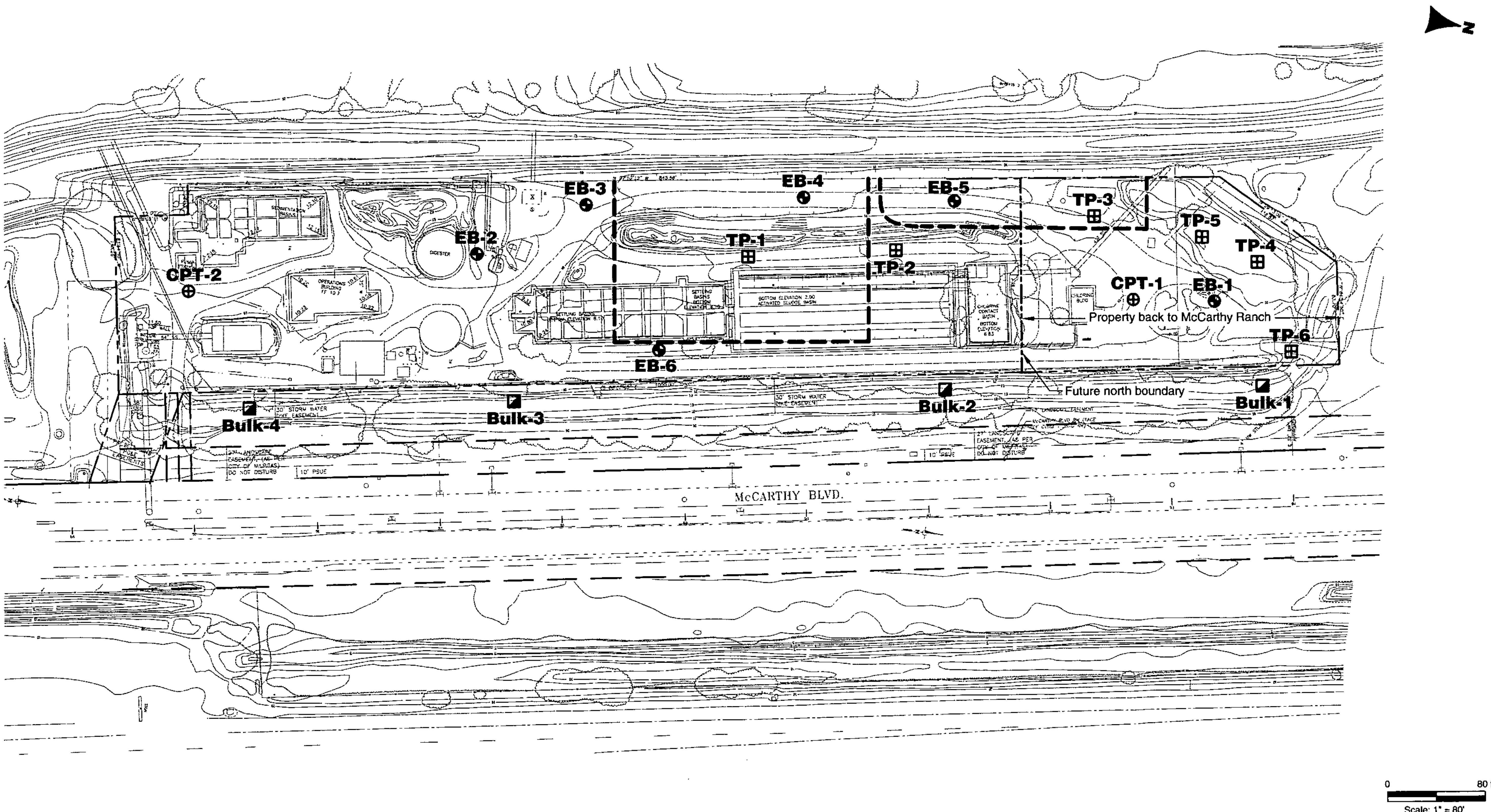

LEGEND

- Approximate location of exploratory boring
- Approximate location of cone penetration test
- Approximate location of test pit
- Approximate location of bulk sample
- Limits of sludge lagoons as determined by McCulley, Frick & Gilman, Inc., 1999; aerial photos; and subsurface data

Base by West Yost & Associates, undated.

SITE PLAN
MAIN SEWER PUMP STATION
Milpitas, California

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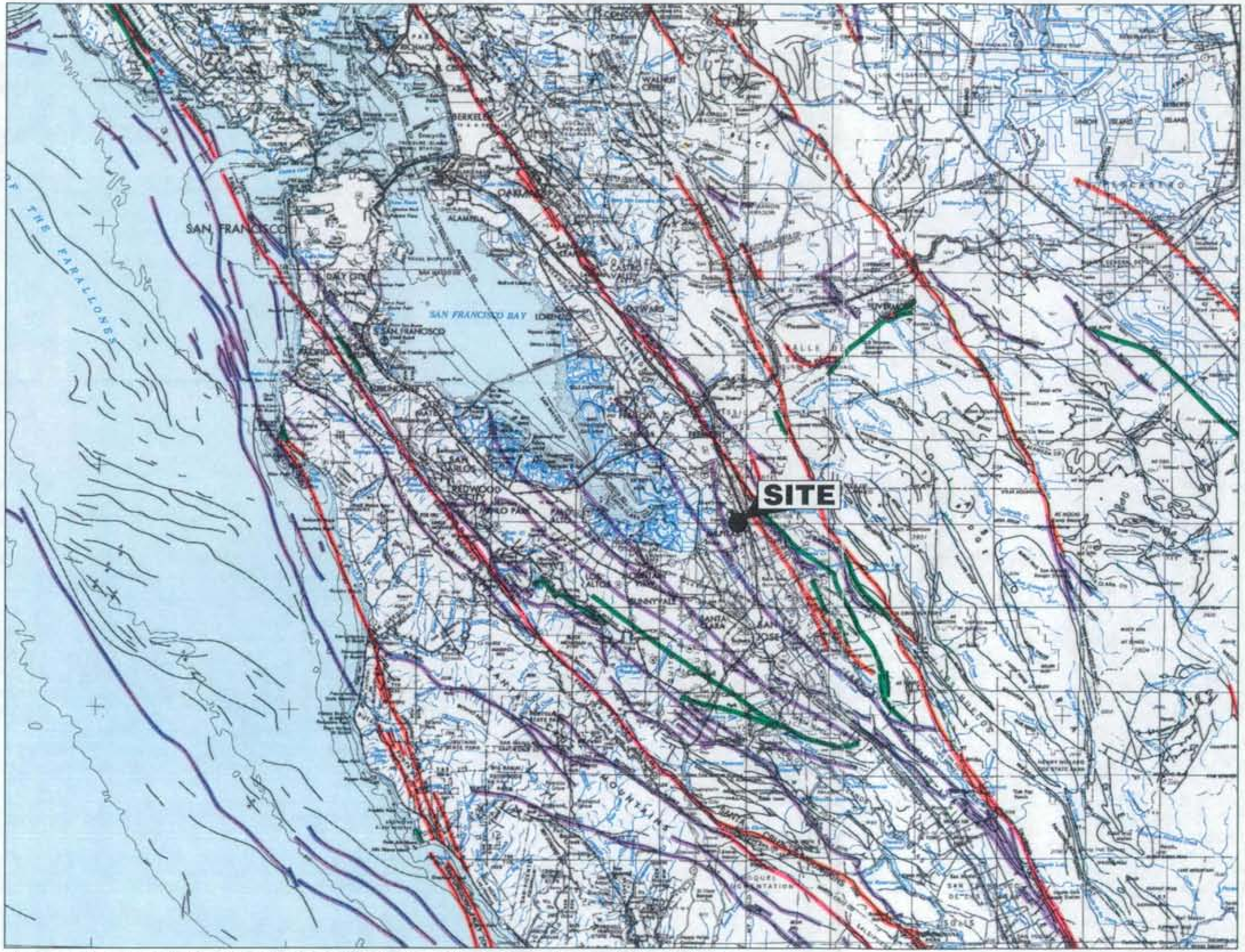
Environmental/Geotechnical/Engineering Services

FIGURE 2
869-4

6/04 EB



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore	DESCRIPTION
Quaternary	Late Quaternary	Historic		~		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200	~		Displacement during Holocene time.
		Pleistocene	10,000	~		Faults showing evidence of displacement during late Quaternary time.
	Early Quaternary	Pleistocene	700,000	~		Quaternary (undifferentiated) faults – most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary		Pliocene	2,000,000			
		Miocene	5,000,000	~		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122–A1–TF–250–00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120–A1–TF–250–00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000–meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Division of Mines and Geology, James F. Davis, State Geologist

REGIONAL FAULT MAP

MAIN SEWER PUMP STATION

Milpitas, California

LOWNEY ASSOCIATES

Environmental/Geotechnical/Engineering Services

FIGURE 3

869-4

6/04 EB

APPENDIX A

FIELD INVESTIGATION

Subsurface exploration was performed on May 20, 2004 using four-wheel rubber-tired backhoe equipment to investigate and sample the subsurface soils. Six exploratory test pits were excavated to depths ranging from 4½ to 7 feet. Further field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling and Cone Penetration Test (CPT)] equipment. Six 8-inch-diameter exploratory borings were drilled on May 26, 2004, to a maximum depth of 44 feet; two CPTs were advanced on May 28, 2004, to a maximum depth of 45 feet. The approximate locations of the test pits, exploratory borings, and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil and CPT interpretations, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the borings were determined by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected Standard Penetration Test measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * * *

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE)	GRAVELS (MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE)	CLEAN GRAVELS (Less than 5% Fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE)	CLEAN SANDS (Less than 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt-mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE)	SILTS AND CLAYS (LIQUID LIMIT IS LESS THAN 50 %)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (LIQUID LIMIT IS GREATER THAN 50 %)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 | 0.4 | 2 | 5 | 19 | 76mm

GRAIN SIZES

TERZAGHI SPLIT SPOON STANDARD PENETRATION

MODIFIED CALIFORNIA

D&M UNDERWATER SAMPLER

SHELBY TUBE

NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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FIGURE A-1

EXPLORATORY BORING: EB-1

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-26-04 FINISH DATE: 5-26-04	COMPLETION DEPTH: 44.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

SURFACE ELEVATION: 9 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
9.0	0	**LEAN CLAY (CL)** stiff, moist, gray and brown mottled, some fine sand, low to moderate plasticity, iron staining; Plasticity Index = 13, Liquid Limit = 36	CL	19	20	98	
				18	22	100	
3.5		**SILTY SAND (SM)** loose, moist, brown, fine sand	SM	17	17	102	25
1.3		**SANDY SILT (ML)** soft, moist, gray, fine sand	ML	4	43		
-0.5	10	**FAT CLAY (CH)** soft, moist, dark gray to gray, some organics, high plasticity	CH	5	40	80	
-8.0 ∇		**LEAN CLAY (CL)** stiff, moist, brown, trace fine sand, low to moderate plasticity	CL	13	30	93	
		becomes very stiff, gray and brown mottled		22			
-21.0	30	becomes stiff, dark gray		14			

Continued Next Page

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA CORP.GDT 6/24/04 MV* FLL

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Environmental/Geotechnical/Engineering Services

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-
BORING TYPE: 8 INCH HOLLOW-STEM AUGER
LOGGED BY: ELS
START DATE: 5-26-04 FINISH DATE: 5-26-04

PROJECT NO: 869-4
PROJECT: MAIN SEWER PUMP STATION
LOCATION: MILPITAS, CA
COMPLETION DEPTH: 44.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
-21.0	30	**LEAN CLAY (CL)** stiff, moist, brown, trace fine sand, low to moderate plasticity	CL				
-25.3	35	**POORLY GRADED SAND WITH SILT (SP-SM)** dense, wet, gray, fine to coarse sand, trace fine gravel	SP-SM	34 37	21		5
-28.5	40	**LEAN CLAY (CL)** stiff, moist, brown and gray mottled, some fine sand, low to moderate plasticity	CL	22 20			
-35.0	45	Bottom of Boring at 44 feet					
	50						
	55						
	60						

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA CORP.GDT 6/24/04 MV* FLL

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EXPLORATORY BORING: EB-2

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-26-04 FINISH DATE: 5-26-04	COMPLETION DEPTH: 25.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression. Scale: 1.0, 2.0, 3.0, 4.0

SURFACE ELEVATION: 11 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
11.0	0	**SANDY LEAN CLAY WITH GRAVEL (CL)** stiff, moist, brown, fine to coarse sand, fine to coarse gravel, low plasticity	CL, FILL	16	14	103	
8.8		**SILTY CLAY WITH SAND (CL-ML)** stiff, moist, brown, fine sand, low plasticity	CL-ML	7	16	95	
	5			5	18		
4.0		**SILTY SAND (SM)** loose, moist, brown, fine sand	SM	5	32	88	
3.5		**SILT WITH SAND (ML)** very soft, moist, brown, fine sand, low plasticity	ML				
3.0	10	**PORLY GRADED SAND (SP)** loose, moist, brown, fine sand, some medium sand	SP	10	28		29
-1.0	15	**FAT CLAY (CH)** soft, moist, dark gray to gray, some organics, high plasticity	CH	12	57	69	
	20			9	48	76	
-11.0		**LEAN CLAY (CL)** stiff, moist, brown, trace fine sand, low to moderate plasticity	CL	23			
-14.0	25	Bottom of Boring at 25 feet					
	30						

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 6/24/04 MV' FLL

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EXPLORATORY BORING: EB-3

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-26-04 FINISH DATE: 5-26-04	COMPLETION DEPTH: 22.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 9 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
9.0	0	**LEAN CLAY (CL) [FILL]** hard, moist, brown, some fine sand, low to moderate plasticity	CL, FILL	33	14	100	
		some gray mottles, wood organics, petroleum odor		18			
4.8	5	**SILTY SAND (SM)** loose, moist, olive gray, fine sand, low plasticity	SM	13	20	92	62
2.5		**POORLY GRADED SAND (SP)** loose, moist, gray and brown, fine sand	SP	11	11	97	
	10	wet, fine to medium sand		6	22		4
-2.0		**FAT CLAY (CH)** soft, moist, dark gray to gray, some organics, high plasticity	CH	6	61	63	
	15			6	42	74	
-11.0	20	**LEAN CLAY (CL)** stiff, moist, brown, trace fine sand, low to moderate plasticity	CL	27			
-13.5		Bottom of Boring at 22½ feet					
	25						
	30						

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 6/24/04 MV' FLL

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EXPLORATORY BORING: EB-4

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-26-04 FINISH DATE: 5-26-04	COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 9 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
8.5	0	**LEAN CLAY (CL) [FILL]** hard, moist, brown, some fine sand, low plasticity	CL,FILL	35	15	113	
4.5		some gray mottles, organic content 2.6%		16	18	105	
	5	**SILT WITH SAND (ML) [FILL]** hard to very stiff, moist, gray brown, fine sand, low plasticity	ML,FILL	25			
		organics		11	29	93	
	10	organics		6			
-2.0		**FAT CLAY (CH)** soft, moist, dark gray to gray, abundant silty sand layers, some organics, high plasticity, organic content 2%	CH	5	29	80	
-5.0	15	**FAT CLAY (CH)** soft, moist, dark gray to gray, high plasticity	CH	5	49	73	
				20			
-9.5		**LEAN CLAY (CL)** stiff, moist, gray and brown mottled, trace fine sand, low to moderate plasticity	CL	20	21	106	
-11.5	20	Bottom of Boring at 20 feet					
	25						
	30						

GROUND WATER OBSERVATIONS:

∇: FREE GROUND WATER MEASURED DURING DRILLING AT 9.5 FEET

LA CORP.GDT 6/24/04 MV* FLL

LOWNEY ASSOCIATES
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EXPLORATORY BORING: EB-5

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-26-04 FINISH DATE: 5-26-04	COMPLETION DEPTH: 26.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 10 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
9.5	0	**SANDY LEAN CLAY WITH GRAVEL (CL) [FILL]** hard, moist, brown, fine to coarse sand, fine to coarse gravel, low plasticity	CL, FILL	25	17	112	
8.3		**LEAN CLAY (CL) [FILL]** hard, moist, brown, some fine sand, low plasticity	CL, FILL	19	16	111	
5.5	5	**SILTY CLAY WITH SAND (CL-ML) [FILL]** very stiff, moist, brown, fine sand	CL-ML, FILL	15	18	105	
3.5		**SILT WITH SAND (ML) [FILL]** stiff, moist, brown, fine sand, low plasticity, interbedded with silty sand and sand with silt	ML, FILL	11			
1.0	10	**SILT WITH SAND (ML)** stiff, moist, brown, fine sand, low plasticity, interbedded with silty sand and sand with silt	ML	19	25	98	
-1.0 ▽		**FAT CLAY (CH)** soft, moist, dark gray to gray, some organics, high plasticity	CH	4	52	70	
	15			4			
				9	42	84	
-8.0		**POORLY GRADED SAND WITH SILT (SP-SM)** loose to medium dense, wet, gray and brown, fine to coarse sand, trace fine gravel	SP-SM	17			
	20						
	25			27			
				25			
-16.8 / -17.0		**LEAN CLAY (CL)** very stiff, moist, brown, trace fine sand, low to moderate plasticity	CL				
		Bottom of Boring at 26½ feet					
	30						

GROUND WATER OBSERVATIONS:

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 10.5 FEET

LA CORP.GDT 6/24/04 MV* FLL

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EXPLORATORY BORING: EB-6

DRILL RIG: MOBILE B-	PROJECT NO: 869-4
BORING TYPE: 8 INCH HOLLOW-STEM AUGER	PROJECT: MAIN SEWER PUMP STATION
LOGGED BY: ELS	LOCATION: MILPITAS, CA
START DATE: 5-27-04 FINISH DATE: 5-27-04	COMPLETION DEPTH: 23.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 9 FT. (+/-)

Undrained Shear Strength (ksf): ○ Pocket Penetrometer; △ Torvane; ● Unconfined Compression; ▲ U-U Triaxial Compression (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
9.0	0	**SILTY CLAY WITH SAND (CL-ML)** hard, moist, brown with gray mottles iron staining, some fine sand, low plasticity	CL	24	17	105	
		soft		7			
	5	very soft, wet		11	31	87	
2.5		**POORLY GRADED SAND (SP)** loose, moist, brown, fine sand, trace medium sand	SP	15	12	97	
	10	wet		8	22		4
				4			
-2.5		**FAT CLAY (CH)** soft, moist, dark gray, some silty sand layers, high plasticity, organic content 15%	CH	3	31		
-6.0	15	**POORLY GRADED SAND WITH SILT (SP-SM)** medium dense, wet, dark gray, fine to medium sand, some coarse sand, some fine gravel	SP-SM	17	21		5
				25			
-11.5	20	**LEAN CLAY (CL)** stiff, moist, brown with gray mottles, some fine sand, low to moderate plasticity	CL	16			
-14.5		Bottom of Boring at 23½ feet					
	25						
	30						

GROUND WATER OBSERVATIONS:

NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 6/24/04 MV* FLL

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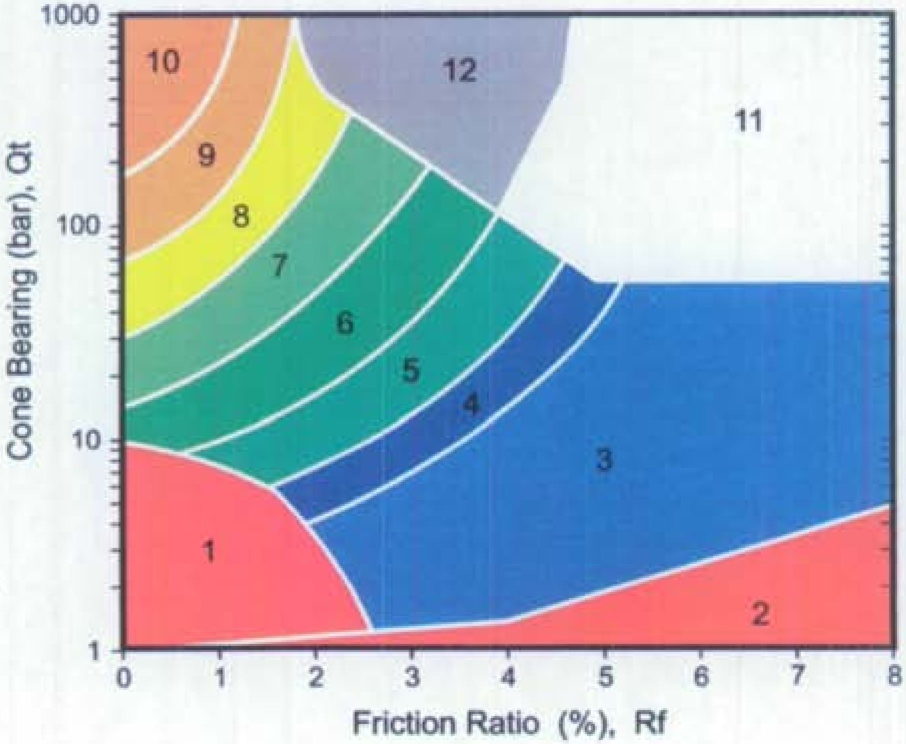
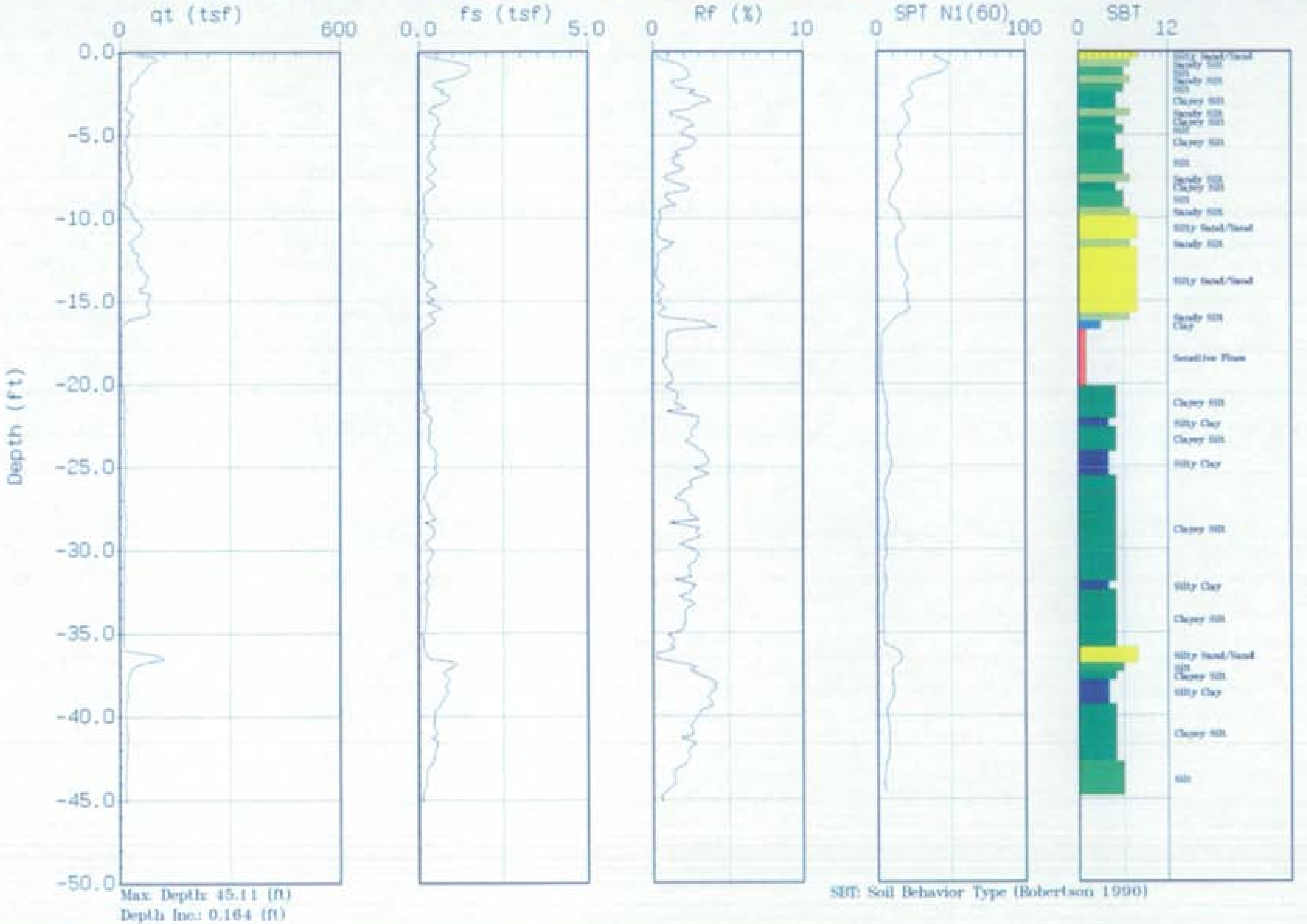


Zone		Q_t / N	Soil Behaviour Type
1		2	sensitive fine grained
2		1	organic material
3		1	clay
4		1.5	silty clay to clay
5		2	clayey silt to silty clay
6		2.5	sandy silt to clayey silt
7		3	silty sand to sandy silt
8		4	sand to silty sand
9		5	sand
10		6	gravelly sand to sand
11		1	very stiff fine grained *
12		2	sand to clayey sand *

* overconsolidated or cemented

Robertson (1990)

KEY TO CONE PENETROMETER TEST



6/04*EB

CONE PENETRATION TEST - CPT-1

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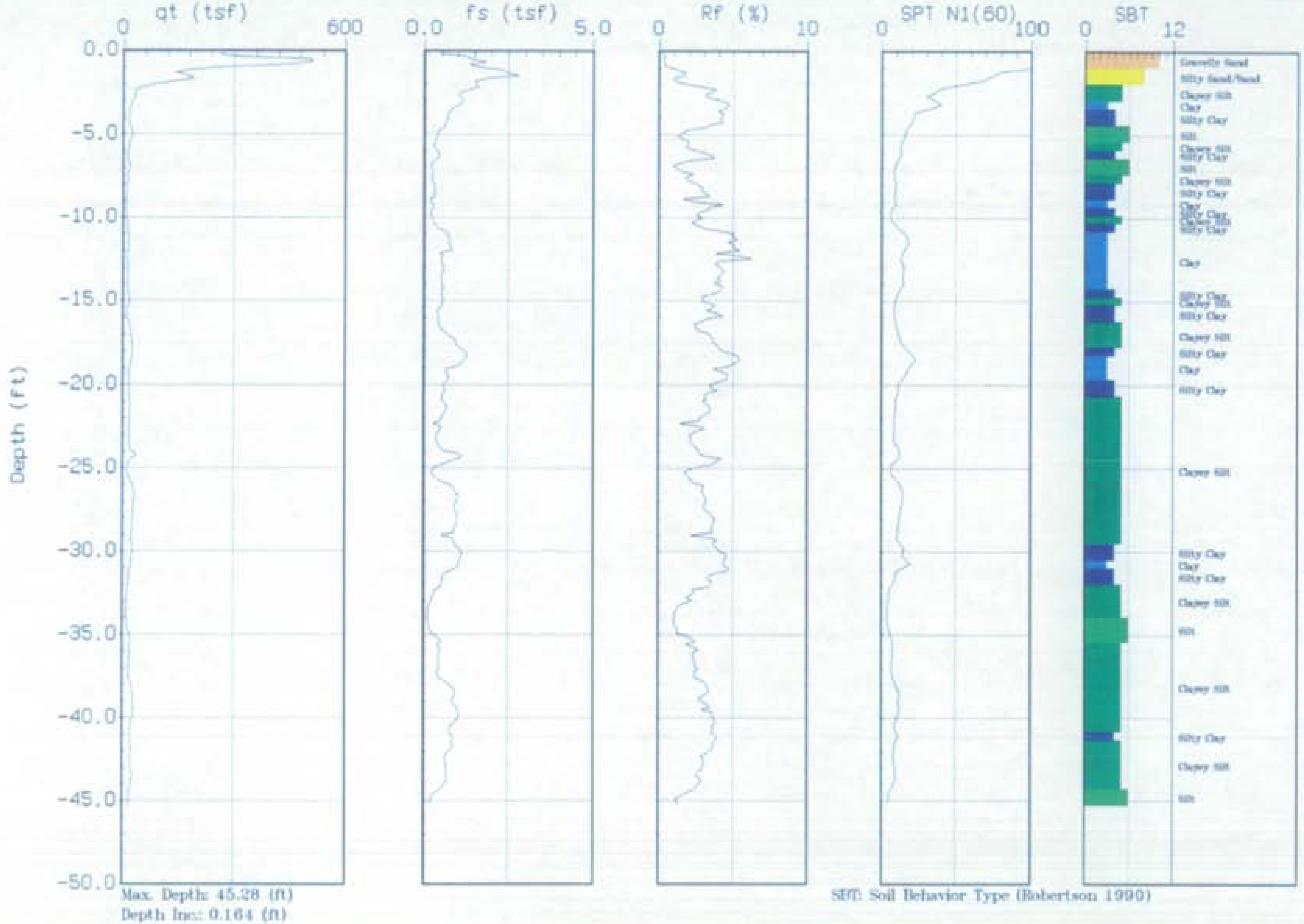
CPT-1
869-4



6/04*EB

CONE PENETRATION TEST - CPT-2

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Environmental/Geotechnical/Engineering Services

CPT-2
869-4

APPENDIX B

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 38 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 34 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are presented on the Plasticity Chart of this appendix and on the log of the boring at the appropriate sample depth.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 7 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Organic Content: Five tests were performed to determine the percent of organic material of selected soil samples (ASTM D 2974). The results of these tests are shown on the logs at the appropriate depths.

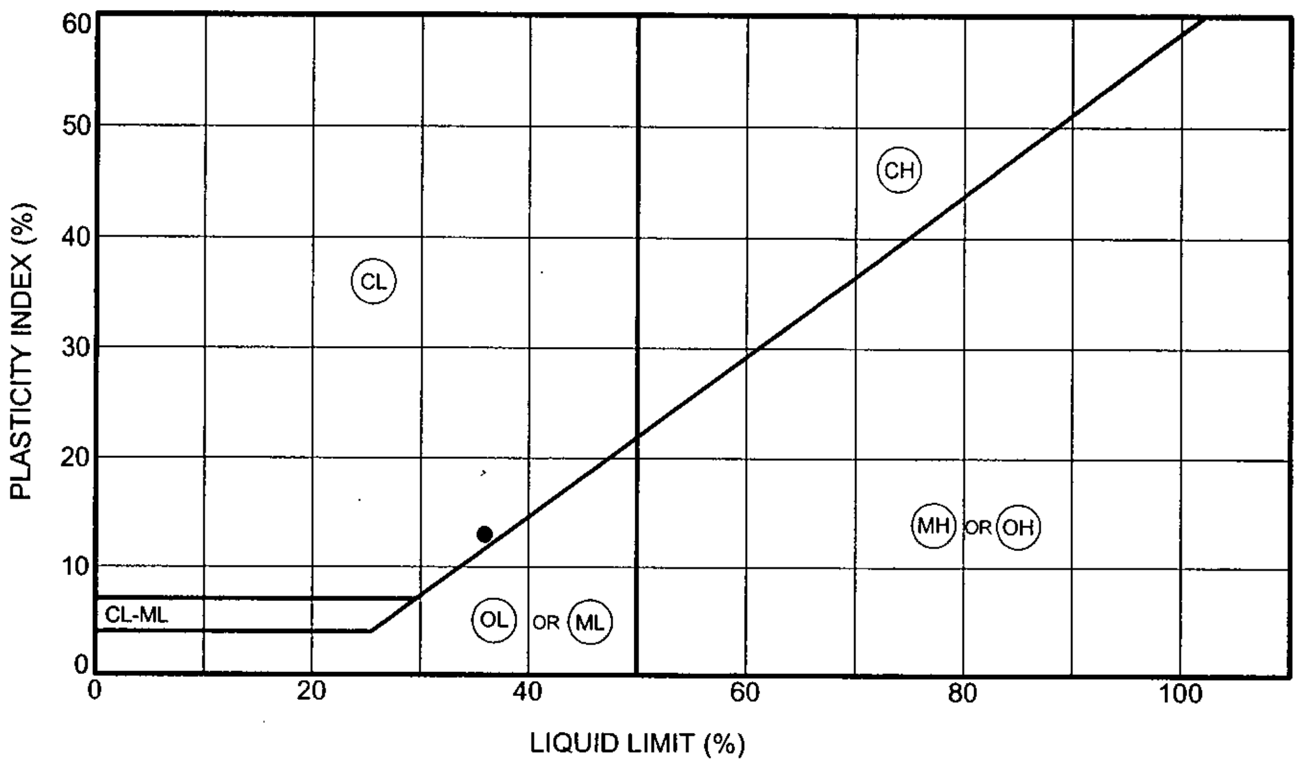
R-Values: R-value (resistance) tests (California Test Method No. 301) were performed on two representative samples of the surface soils at the site to provide data for pavement design. The tests indicated R-values of 44 and 49 for samples collected from EB-1 and EB-2, respectively, at an exudation pressure of 300 pounds per square inch.

Table B-1. Results of R-Value Tests

Sample	Description of Material	Water Content (%)	Dry Density (pcf)	Exudation Pressure (psi)	"R" value	Expansion Pressure (psf)
		13.1	119.3	184	17	0.0
EB-2	Brown Sandy	11.2	123.1	287	48	111.8
Bulk	Silt	9.8	124.9	645	67	232.2
		14.3	116.9	206	18	0.0
EB-1	Brown Silty	13.3	119.2	268	38	60.2
Bulk	Clay	11.9	121.1	417	62	369.8

Soil and Plant Laboratory Tests: Five samples were sent to the Soil and Plant Laboratory for analysis. Four samples were collected from the test pits within the berm along North McCarthy Boulevard and one was collected from Test Pit TP-3, one of the on-site stockpiles. Results of these tests and their soil amendment recommendations are presented in this appendix.

* * * * * * * * * * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-1**	**1.5**	**20**	**36**	**23**	**13**		**LEAN CLAY (CL)**

LA_CORP.GDT 6/24/04 MV* FLL

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PLASTICITY CHART AND DATA

Project: MAIN SEWER PUMP STATION

Location: MILPITAS, CA

Project No.: 869-4

FIGURE B-1

Soil and Plant Laboratory, Inc.
www.soilandplantlaboratory.com

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Santa Clara, CA 95050
408-727-0330 phone
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SANTA CLARA OFFICE
June 11, 2004
Lab No. 50830

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405 Clyde Avenue
Mountain View, CA 94043

LOWNEY ASSOC.
JUN 15 2004
RECEIVED

Attn: Laura Knutson

RE: MILPITAS PUMP STATION – MILPITAS, JOB #869-4

BACKGROUND

The five samples received 6/2 were identified as representing stockpiled site soils that will be used in areas to be landscaped with turf, groundcover, trees and shrubs. The first four samples were designated as from the Main Sewer Pump berm stockpile while the fifth sample was from the Milpitas Sewage onsite stockpile.

SUMMARY

Sample 5 shows the greatest degree of deviation from the other materials in that it contains a greater amount of sodium and this is to a degree that will have a deleterious affect on soil structure. Chemistry and fertility of these can be adjusted appropriately, but even with aggressive amending the fine texture of these soils will result in slow drainage capability and high water retention. Plants susceptible to problems in poorly drained soils should be avoided and irrigation management should be very carefully monitored.

ANALYTICAL RESULTS

Particle size data show all falling in the clay classification by USDA standards. Sample 5 contains a little less clay than the others, but still with combined silt plus clay in this range of 75 to 80%, these are all very fine textured materials. In no case is organic content high enough to have a beneficial affect on soil structure. Infiltration rates are estimated at a very slow 0.14-inch per hour and could even be slower in soil represented by sample 5 due to the impact of the sodium imbalance.

Reaction values are neutral to slightly alkaline and with no significant amount of lime present this is suitable for a wide variety of plants. While this present reaction is not problematic for most plants, including a modest amount of sulfur in the amendment program will move this to a slightly acidic level that will help maximize overall nutrient availability. Boron is safely low in all. In 1 through 4 salinity and sodium are safely low and the SAR values show soluble sodium sufficiently balanced by calcium and magnesium. Salinity of sample 5 is slightly high as a reflection of higher sodium than found in the other materials. Sodium accounts for about half of the salinity level. This is resulting in a sodium adsorption ratio that is high enough to adversely affect structure. When the appropriate amendments have been added salinity will initially be high enough that particularly thorough irrigations will be required in order to wash some of the soluble sodium out of the immediate root zone.

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Nutrient availability data show nitrogen low except for it being adequate in 5. Phosphorus is variable being deficient in 1 and 2, fair in 3 and 5 and adequate in 4. Potassium levels range from deficient to fair at best and some interference with potassium nutrition is expected from the uniformly high level of magnesium. Sulfate levels are fair and calcium is lower than desired relative to the abundance of magnesium.

RECOMMENDATIONS

A single amendment program is suggested in order to simplify matters. While this exceeds the minimum nutritional needs in some cases it will not create any problem. The surplus nutrients will just result in better reserve levels. While sample 5 requires the addition of gypsum to bring about a sodium balance it is also needed in the other soils to bring calcium availability into better balance with the high magnesium level.

Once these soils are in place the following materials should be uniformly spread and incorporated with the top 6 inches:

Amount / 1000 Square Feet	
6 cubic yards	Nitrogen Stabilized Organic Amendment
12 pounds	6-20-20 Commercial Fertilizer
8 pounds	Potassium Sulfate (0-0-50)
10 pounds	Soil Sulfur
70 pounds	Agricultural Gypsum

The amendment rate suggested is based on an organic matter content of 250 pounds per cubic yard of amendment and this may be adjusted depending on the organic content of the amendment selected. If nutrient-rich compost is used, then it may be suitable to modify the fertilizer rate. Sulfur and gypsum would still be needed.

Surface soil that has been amended in the above manner may be used as the top foot of backfill around the sides of the rootball of trees and shrubs. Deeper backfill required around the sides of the rootball of 24-inch box or larger material should not contain the extra organic matter or sulfur but should be blended with 1 pound 6-20-20, 1-1/2 pounds iron sulfate and 3 pounds gypsum per cubic yard. Soil immediately below the rootball should be left undisturbed to provide support but the bottom around the sides should be dusted with gypsum and cultivated to improve porosity. Slow release fertilizer tablets may be placed in the backfill at label rates

Initial maintenance fertilization may rely on a program that applies ammonium sulfate (21-0-0) at a rate of 5 pounds per 1000 square feet. The first application should be 30 days after planting with retreatment scheduled at 45 to 60-day intervals. Alternatively, slow release Sulfur Coated Urea (32-0-0) may be

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applied at an 8-pound rate with refertilization scheduled at 3-month intervals. Once the landscape has become well established the frequency of fertilization may be decreased depending on color and rate of growth desired. In the fall and spring substitute a complete fertilizer such as 16-6-8 to help insure continuing adequate phosphorus and potassium.

Jim West

JIM WEST

E-mail 4 pages and mail hard copy. / jr

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COMPREHENSIVE SOIL ANALYSIS
(A05-1, A05-2 or A05-3)

Santa Clara Office
Lab No. 50830
MILPITAS PUMP STATION
Job No. 869-4

Samples Rec'd: 6/2/04

Sample #	Half Sat%/ TEC	pH/ Qual Lime	ECe	NO3 N	NH4 N	PO4 P	K	Ca	Mg	Cu	Zn	Mn	Fe	Organic % dry wt.	Sample Description & Log Number
				------------Parts Per Million Parts Dry Soil-------------											
1	24	7.3	0.8	13	7	16	170	3840	1450					2.0	TP#1 Main Sewer Pump Berm Stockpile
	318	None		0.4		0.6	0.4	0.9	2.6						0.8404-A10975 20 4
2	25	7.4	0.9	19	7	16	150	3830	1440					1.8	TP#2 Main Sewer Pump Berm Stockpile
	314	None		0.5		0.5	0.4	0.9	2.5						0.8704-A10976 20 4
3	24	7.0	0.8	8	12	19	270	3660	1470					2.9	TP#3 Main Sewer Pump Berm Stockpile
	318	None		0.4		0.7	0.7	0.8	2.6						0.8404-A10977 20 4
4	26	7.2	0.8	18	9	30	220	3560	1570					2.1	TP#4 Main Sewer Pump Berm Stockpile
	313	None		0.5		1.0	0.6	0.8	2.6						0.9004-A10978 20 4
5	26	7.4	3.5	61	7	21	160	2790	1150					1.8	TP#5 Milpitas Sewage On Site
	257	None		1.3		0.7	0.4	0.7	2.1						Stockpile 0.904-A10979 20 4

Sample #	Ca me/l	Mg me/l	Na me/l	K me/l	B ppm	SO4 me/l	SAR	Gravel: Coarse 5-12	Gravel: Fine 2-5	Sand: Very Coarse 1-2	Sand: Coarse 0.5-1	Sand: Med. to V. Fine 0.05-.5	Silt .002-.05	Clay 0-.002	USDA Soil Classification
	--------Saturation Extract Values---------							Percent of Sample Passing 2 mm Screen							
1	3.2	2.7	2.5	0.1	0.23	1.3	1.5	0.7	1.0	0.8	1.5	15.5	32.0	50.2	Clay
2	3.5	3.0	2.2	0.1	0.24	1.5	1.2	1.1	0.8	0.8	1.4	17.3	32.3	48.2	Clay
3	2.9	2.6	3.6	0.2	0.36	1.2	2.2	1.5	0.9	0.8	1.5	17.2	32.3	48.2	Clay
4	3.6	3.6	2.0	0.2	0.26	1.3	1.1	1.1	1.1	1.2	1.6	13.0	34.0	50.2	Clay
5	7.2	7.4	19.9	0.2	0.65	9.0	7.4	8.7	4.9	2.6	2.8	19.8	33.6	41.2	Clay

6/ 8/04

Sufficiency factor (1.0=sufficient for average crop) below each nutrient value. N factor based on 200 ppm constant feed. SAR = Sodium adsorption ratio. Half Saturation %=approx field moisture capacity. Nitrogen(N), Potassium(K), Calcium(Ca) and Magnesium(Mg) by sodium chloride extraction. Phosphorus(P) by sodium bicarbonate extraction. Copper(Cu), Zinc(Zn), Manganese(Mn) & Iron(Fe) by DTPA extraction. Sat. ext. method for salinity (ECe as dS/m), Boron(B), Sulfate(SO_4), Sodium(Na) and SAR. TEC(listed below Half Sat) = Est.Total Exchangeable Cations(meq/kg). Gravel fraction expressed as percent by weight of oven-dried sample passing a 12mm(1/2 inch) sieve. Particle sizes in millimeters.

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